

# Seismic Retaining Wall Failure

Best Practices in Dam and Levee Safety Risk Analysis

Part E – Concrete Structures

Chapter E-7

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US Army Corps  
of Engineers®



# Outline

- Objectives
- Key concepts
- Potential failure modes
- Case histories
- Key inputs and considerations
- Seismic earth pressures
- Centrifuge tests and experience
- Finite element studies
- Event tree evaluation



# Objectives

- Understand the mechanisms that affect retaining wall failure under seismic loading
- Understand how to construct an event tree to represent potential seismic retaining wall failure
- Understand how to estimate event tree probabilities and probability of seismic retaining wall failure



# Key Concepts

- This potential failure mode typically relates to gated spillway crest structures at embankment dams where spillway walls retain the embankment and the pool
- The evaluation deals with the seismic response of reinforced concrete structures typically with loading from retained soil
- Methods to estimate seismic earth pressures and wall response are therefore important
- Most walls were not designed for these loads, but have reserve capacity
- Typically not an issue for uncontrolled spillways (without gates and water stored against the crest structure, uncontrolled release not likely to result from wall failure)
- Counterforted walls present a special case due to their prevalence and the multiple potential failure modes associated with these types of walls



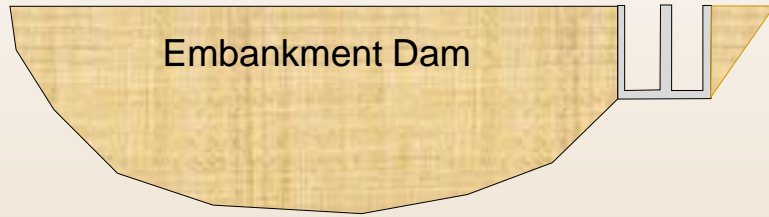
# Potential Failure Modes



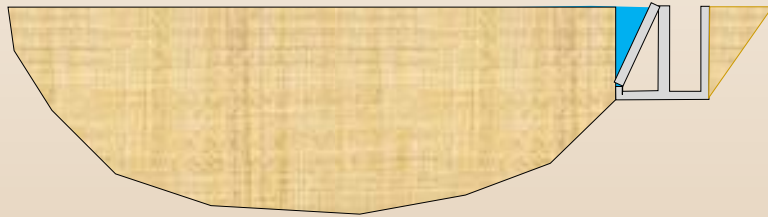
# Seismic Spillway Wall Potential Failure Modes

- Wall failure can occur a number of different ways:
  - Wall collapses (overturns from bearing capacity failure at toe, slides along unreinforced lift joint), or fails in shear, failing adjacent gate
    1. Wall loses structural integrity
    2. Wall collapses inwards
    3. Adjacent gate fails
    4. Uncontrolled release through spillway bay
  - Wall deflects excessively and damages adjacent gate
    1. Wall deflects excessively
    2. Unanticipated load on gate structural member(s)
    3. Gate buckles
    4. Uncontrolled release through spillway bay
  - Wall deflects or fails creating seepage path
    1. Wall deflects sufficiently to create a gap between wall and adjacent embankment
    2. Seepage path established through gap
    3. Progressive erosion of adjacent embankment
    4. Breach of embankment

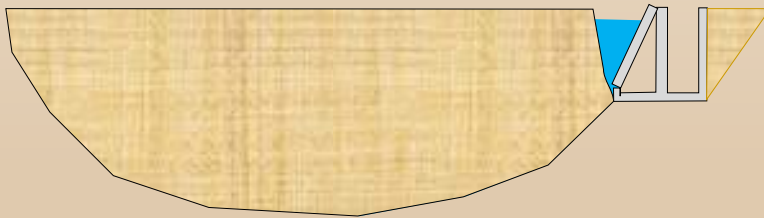
# Seismic Spillway Wall PFM



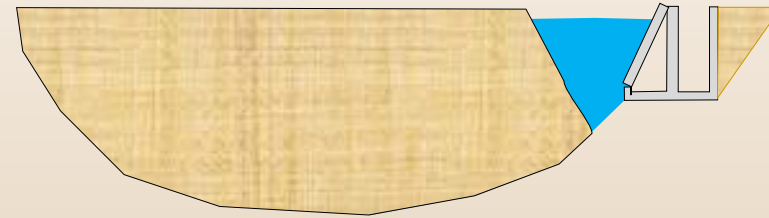
1 - Original Embankment and Gated Spillway



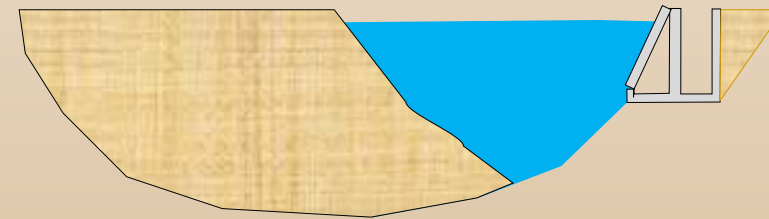
2 – Earthquake damages spillway wall producing upstream to downstream seepage path



3 – Embankment starts to scour



4 – Embankment continues to scour and cannot be stopped



5 – Embankment fails

# Case Histories





# Oh-Kirihata Dam



- Earthquake generated by Futagawa fault rupture through western side of reservoir in 2016.
- Significant damage and offsets to the spillway, including failure of the left retaining wall, shown here.
- Reservoir was full at time of quake, but was drained rapidly, so no breach of the reservoir

# Shi-Kang Dam

- Shi-Kang Dam is a buttress gravity dam located on the Tachia River (covered in other presentations)
- Located about 30 miles north of the epicenter of the Chi-Chi earthquake (9/21/99)
- Differential ground movement through spillway was 29 feet
- PHA – 0.6g near dam
- Spillway chute wall panel failed during 1999 earthquake
- Failure appears to be a shear failure through the counterforts
- No specific details are available for the structure
- But not a water retaining structure, so resulted in no loss of pool – still, an interesting case

# Shi-Kang Dam Spillway Wall





# Austrian Dam Spillway

- Austrian Dam is a 200-foot high embankment dam constructed on Los Gatos Creek, near Los Gatos CA
- Concrete spillway located on right abutment of dam
- Austrian Dam was subjected to Loma Prieta earthquake on 10/17/89
- Estimated that PHA at site was up to 0.6g
- Austrian Dam settled and spread – max settlement = 2.8 ft
- Spillway damage
  - cutoff walls were loaded and displaced
  - chute elongated about 1 foot as a result of embankment deformation
  - Up to 6 inch voids created upstream of cutoff walls
  - chute walls deflected inward
  - potential seepage path created (but reservoir was low at the time of the earthquake)

# Austrian Dam Spillway





# Key Inputs and Considerations

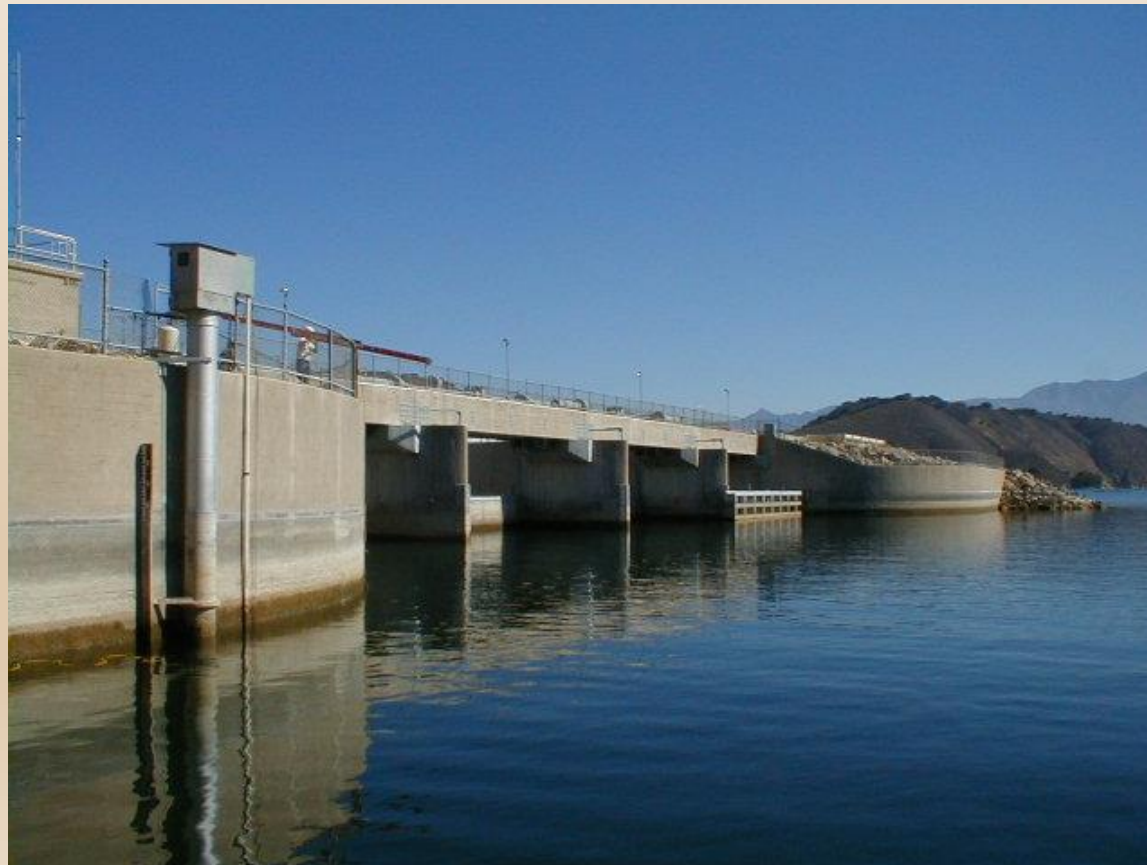


# Key Inputs – Seismic Wall Evaluation

- Reservoir Water Surface Elevation and Static Gate Loads
  - Hydrodynamic Loads from Gates
  - Wall Geometry/Properties
  - Reinforcing
  - Moment Capacity
  - Shear Capacity
  - Seismic Hazard
  - Spillway Bridges/ Hoist Decks
  - Wall Backfill/ Seismic Earth Pressures
- Reinforced Concrete Failure Mechanisms – Covered in separate chapter

# Spillway Hoist Deck/Bridges

- Bridges are typically provided across the top of spillway crest structures – hoist decks and highway bridges
- Bridges may serve as struts for the top of walls but this needs to be verified

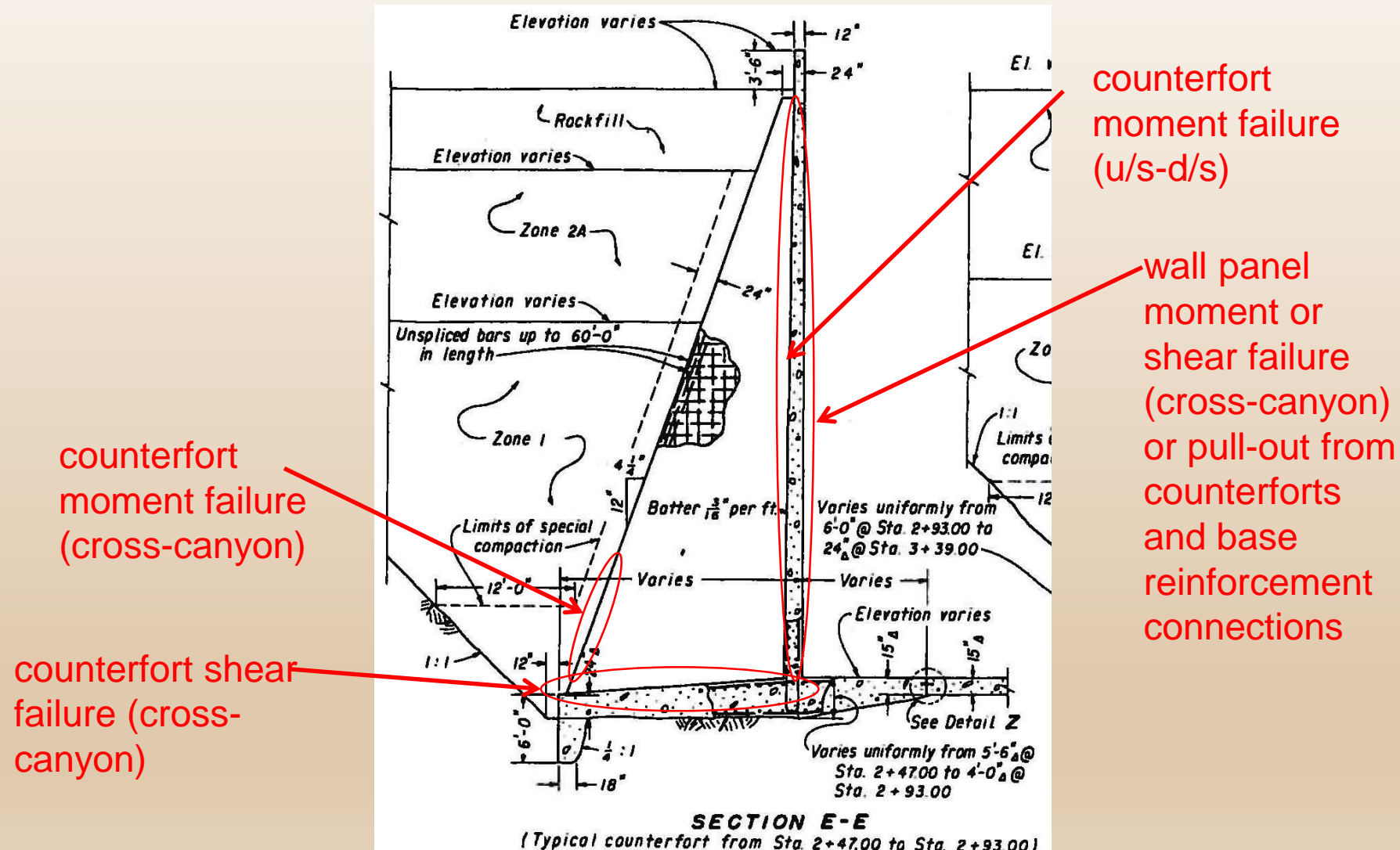


# Bracing by Spillway Gates



- Spillway gates may add some bracing to adjacent walls
- However, seals and skin plate form the initial contact with little resistance
- Significant deflection may be needed to mobilize the strength of the more robust members
- Gate bracing is likely limited and could buckle from wall loads
- This must be considered when assessing resistance from gates

# Counterforted Wall – Failure Mechanisms





# Seismic Earth Pressures



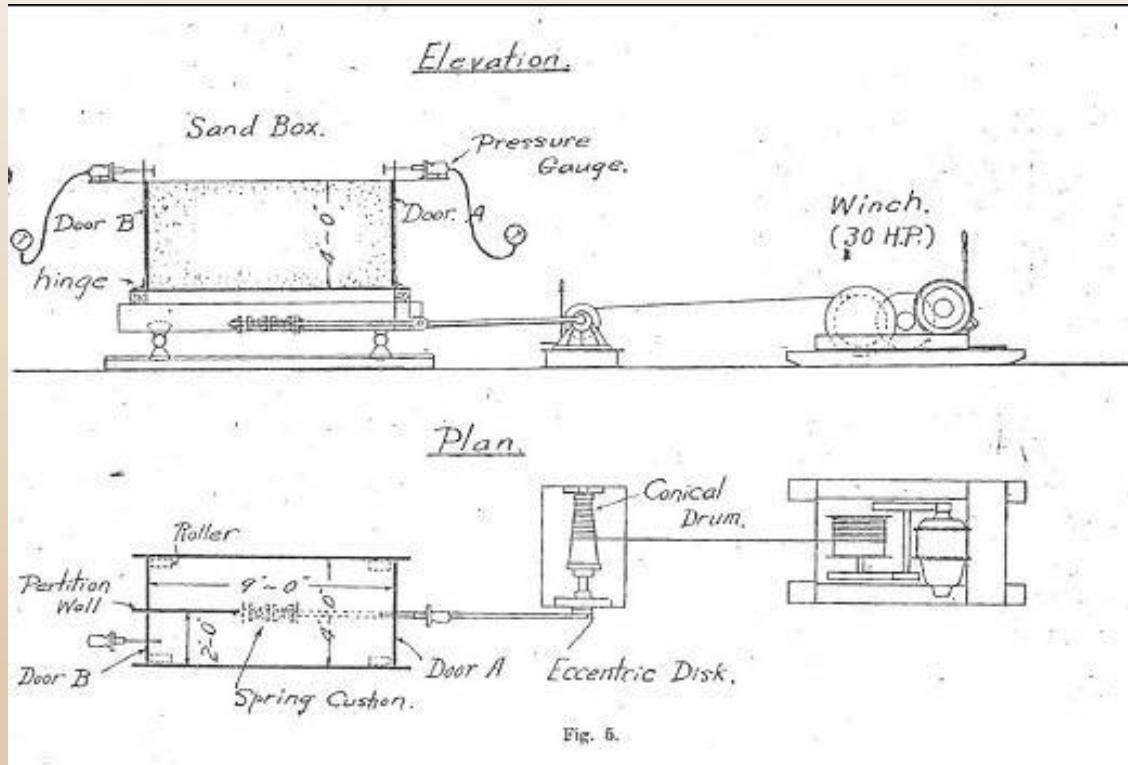
# Seismic Earth Pressure

- Seismic earth pressure is the critical loading mechanism for spillway walls (in combination with static earth pressures – see chapter)
- Related to interaction of spillway wall and backfill
- Affected by spillway crest structure foundation (rock or soil)
- Seismic soil loadings are related to earth pressure theory and the state of the wall backfill prior to and during the earthquake
- Typically Mononobe-Okabe or Wood's solutions are used for screening purposed
- Start with Mononobe-Okabe, but if you have a case where the equation “blows up” (described later), then go to Woods solution
- Finite element analyses reserved for critical cases when evaluating potential cracking and damage

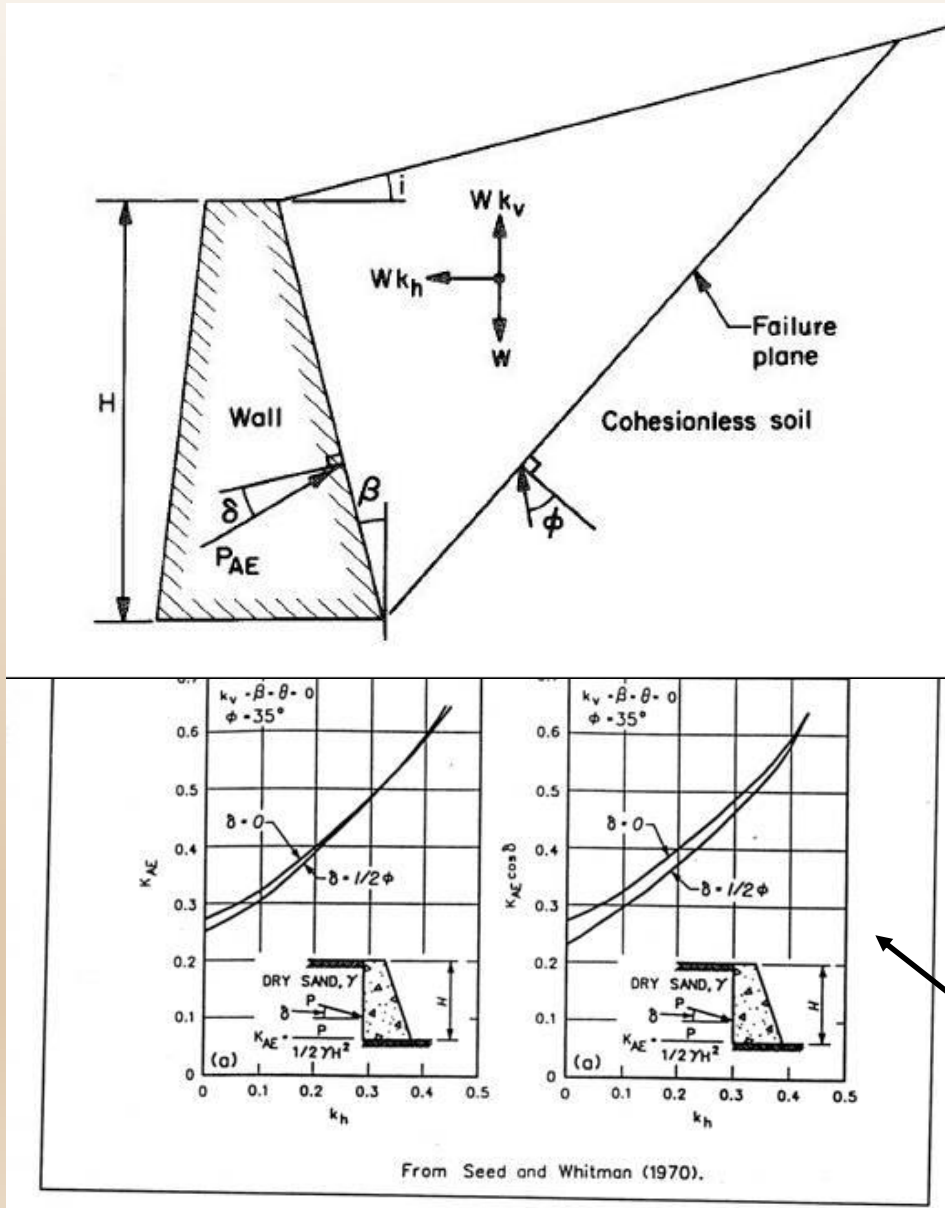


# Seismic Earth Pressure

- Mononobe-Okabe • Mononobe-Matsuo 1929 test in Japan
  - Rigid, small scale, 1g shake table
  - Box (9'Lx4'Wx4'H) filled with loose dry sands on rollers
  - Winch driven by 30 HP electric motor
  - Horizontal simple harmonic motion
  - Hydraulic pressure gauges mounted on top to measure earth pressures



# Mononobe-Okabe Relationship



$$P_{AE} := \frac{1}{2} \cdot \gamma \cdot H^2 \cdot (1 - k_v) \cdot K_{AE}$$

$\gamma$  = unit weight of the soil

$H$  = height of the wall

$k_v$  = vertical wedge acceleration divided by  $g$

Can be used with probabilistic limit state analyses to estimate probabilities

Values of  $K_{AE}$  – includes both static and dynamic earth pressure effects. For moment calculations the two components act at different locations so separate.

# Mononobe-Okabe Assumptions

- Yielding wall with active pressures
- Cohesionless backfill
- Soil satisfies Mohr-Coulomb failure criterion
- Failure plane in backfill occurs along inclined angle and passes through the toe of the wall
- No liquefaction
- Soil wedge behaves as a rigid body and accelerations are constant throughout the mass
- Backfill is completely above or completely below the water table
- Limitation -  $\phi \geq \psi$  (equation blows up)

$$\psi := \operatorname{atan}\left(\frac{k_h}{1 - k_v}\right)$$

$\phi$  = angle of internal friction of the soil

$k_h$  = horizontal wedge acceleration divided by  $g$

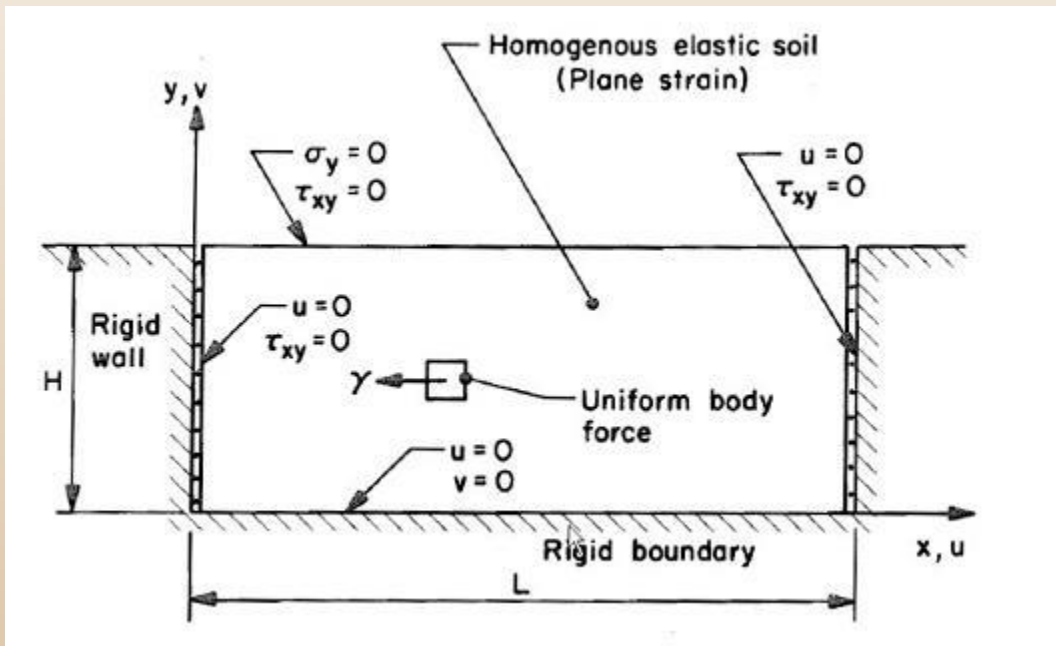
$k_v$  = vertical wedge acceleration divided by  $g$

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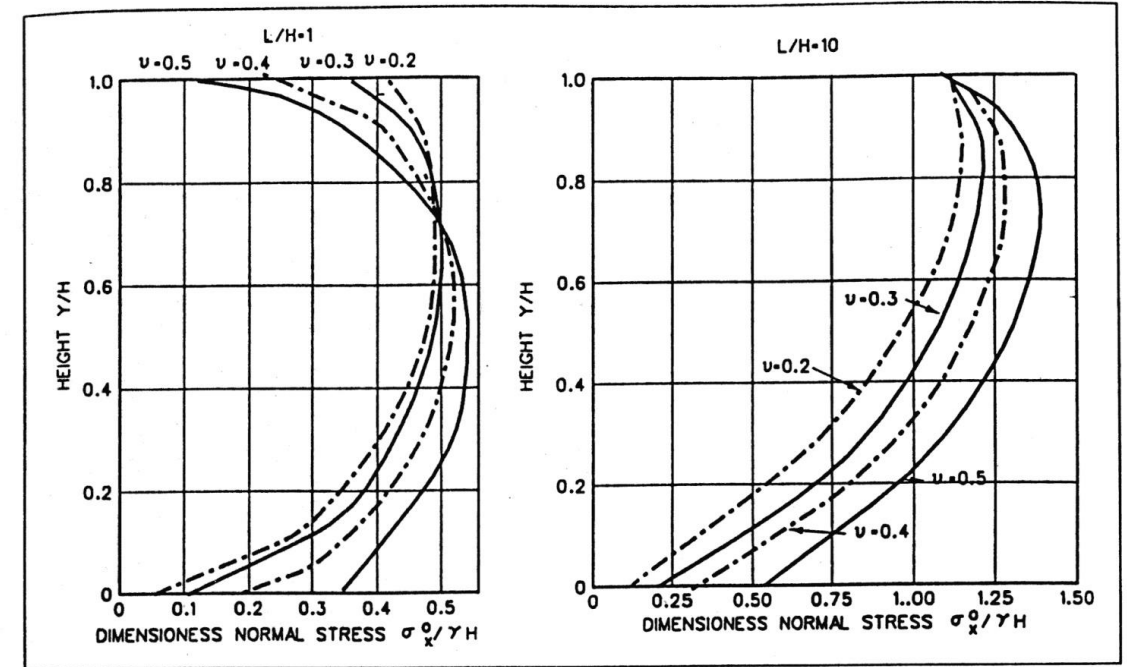
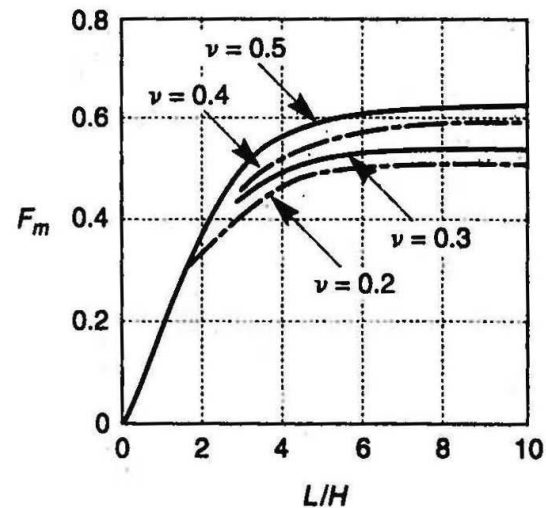
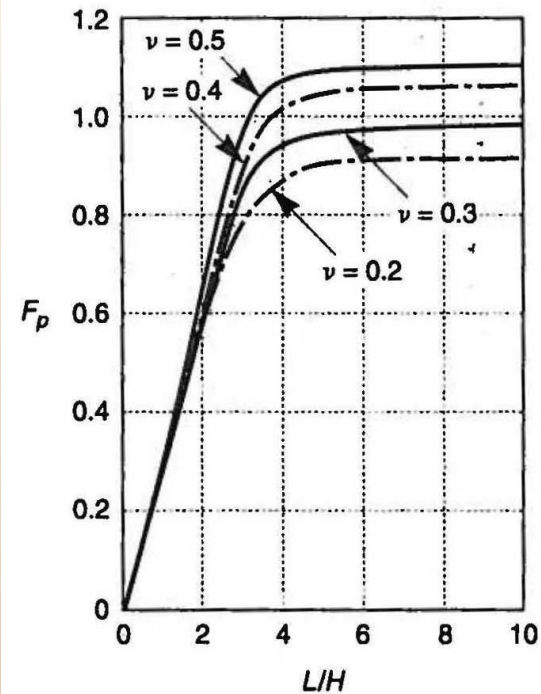


# Wood's Solution



- Elastic method developed in 1973
- Rigid, non-yielding walls
- Displacements generate soil stresses in the elastic range
- Elastic wave solutions
- Upper bound  $\cong 2$  to  $3 \times M-O$
- Dynamic earth pressures must be added to static earth pressures
- Function of soil Poisson's ratio
- Function of  $L/H$
- Not limited for large response accelerations
- Shaking frequency  $\ll$  fundamental backfill frequency
- Normalized solutions
- Generally conservative

# Wood's Solution



$\Delta P_{AE} = \gamma H^2 k_h F_p$  – dynamic thrust

$\Delta M_{AE} = \gamma H^3 k_h F_m$  – dynamic overturning moment

Conservative Parabolic Stress Distribution -  
 $\Delta P_{AE}$  acts @ 0.55 – 0.65H

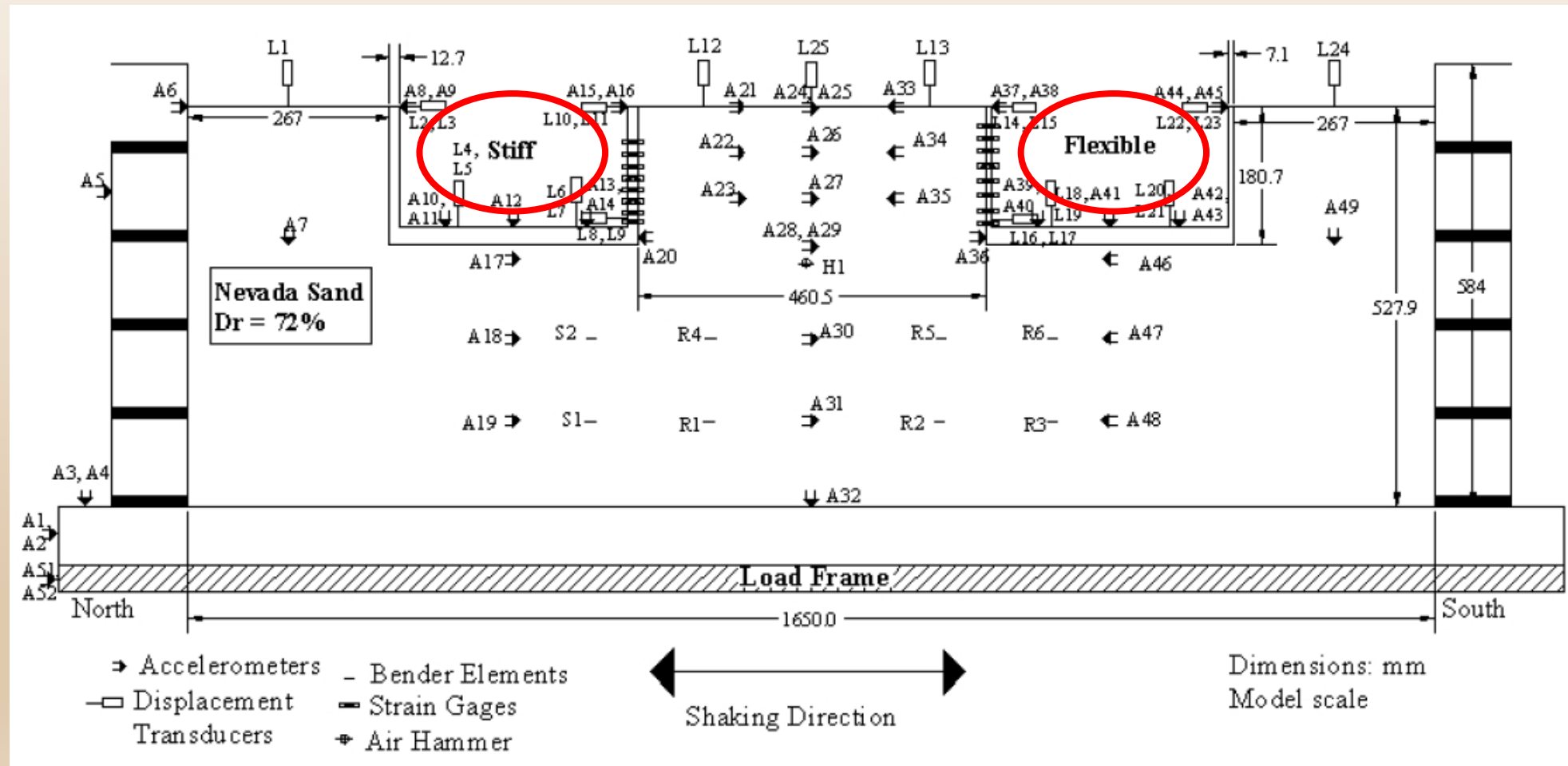
# Centrifuge Tests and Experience



# Research by Sitar and Al-Atik

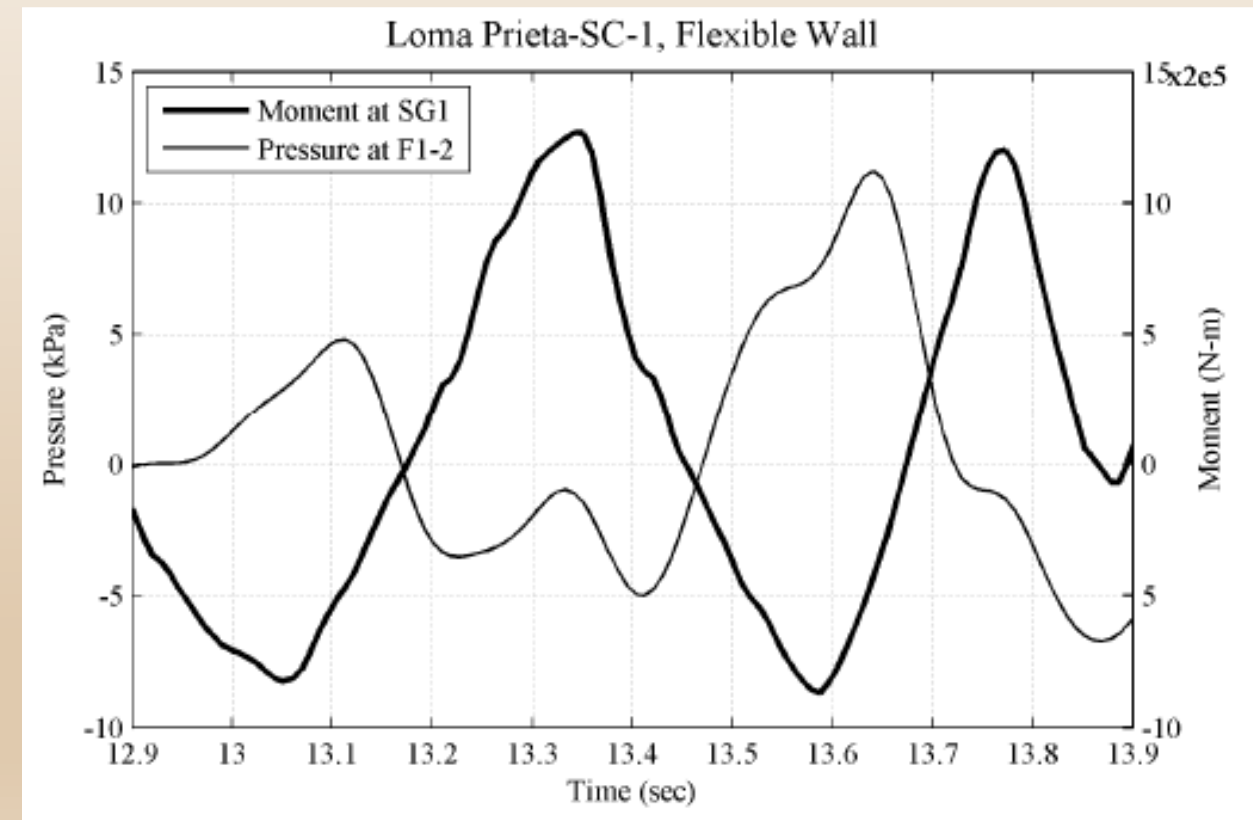
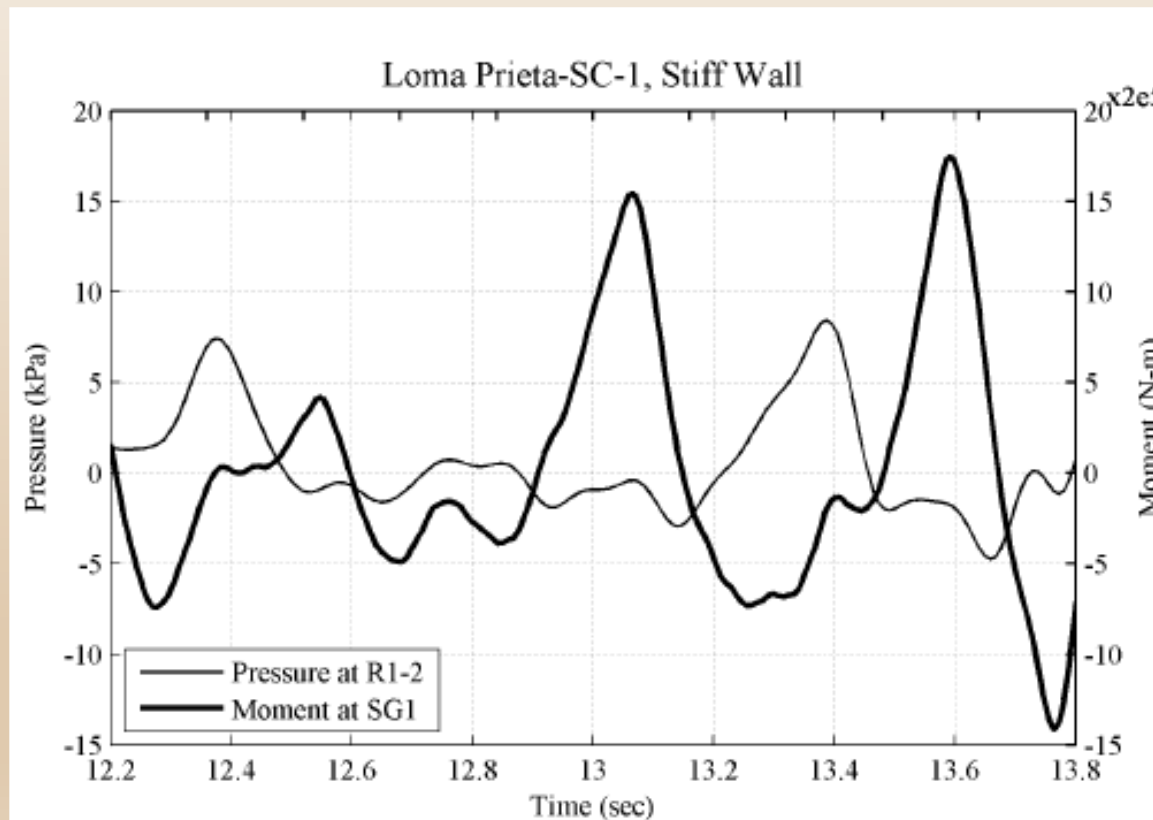
- Centrifuge modeling and numerical modeling of U-shaped retaining walls were conducted with sand backfill
- Models were subjected to ground motions and earth pressures and moments in the wall were measured
- Good historic performance of reinforced concrete walls during earthquakes was also documented
  - Walls retaining dry soils, dense soils, and clayey soils performed very well for PHA as high as 0.5 to 0.6 g (e.g. the Los Angeles Floodway, 1971 San Fernando earthquake)
  - Loose cohesionless saturated soils subject to liquefaction could cause problems for retaining structures

# Centrifuge Layout

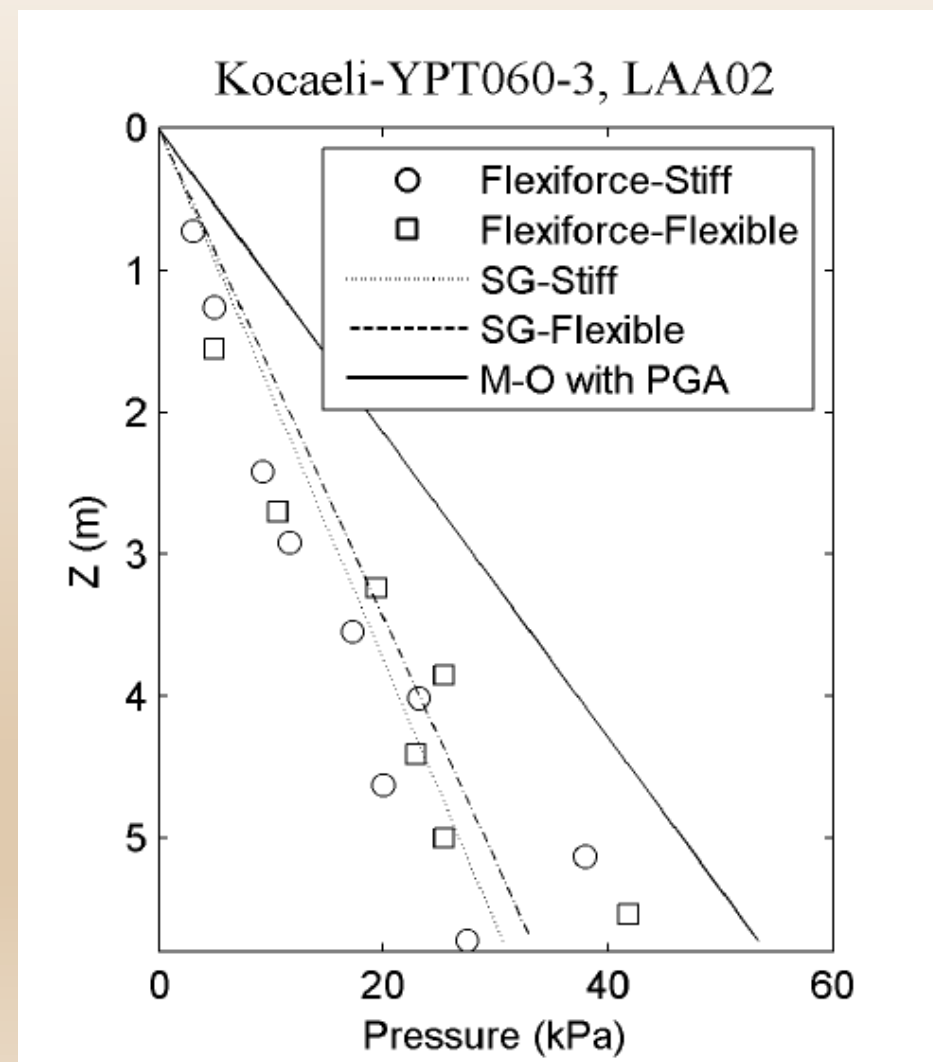
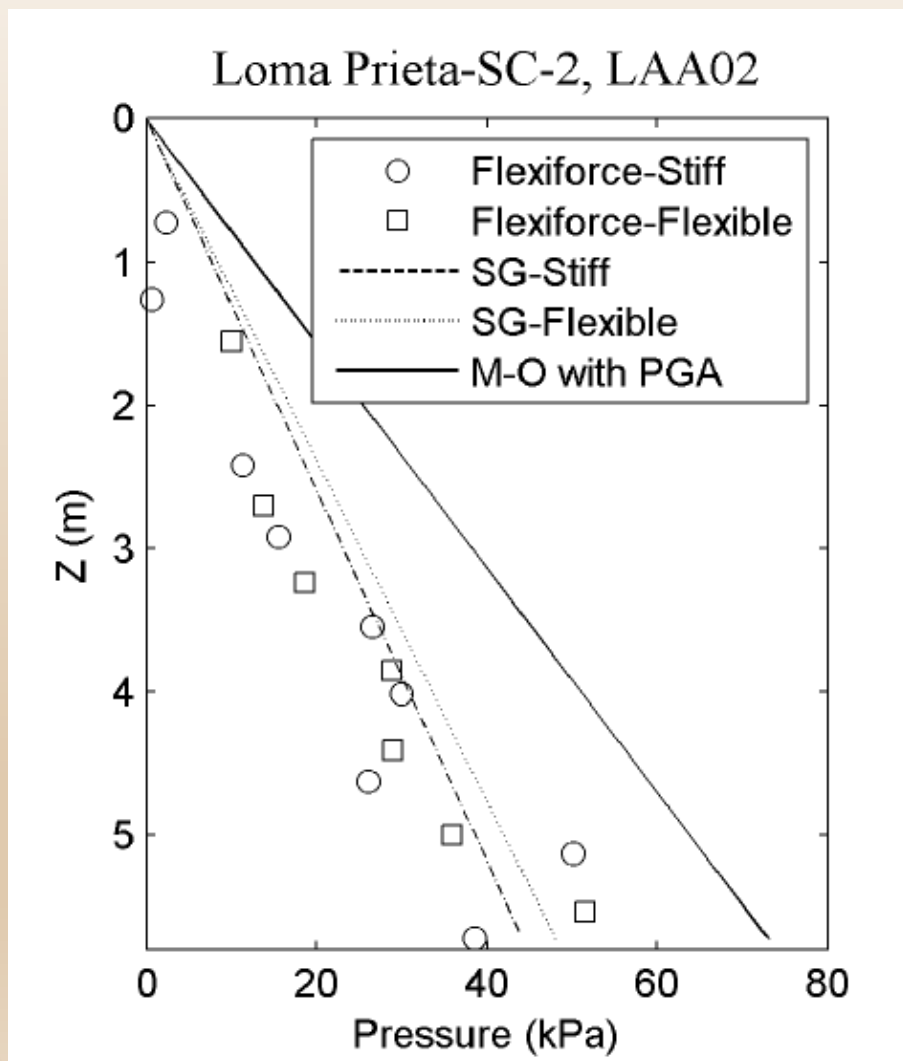




# Centrifuge Results



# Centrifuge Results

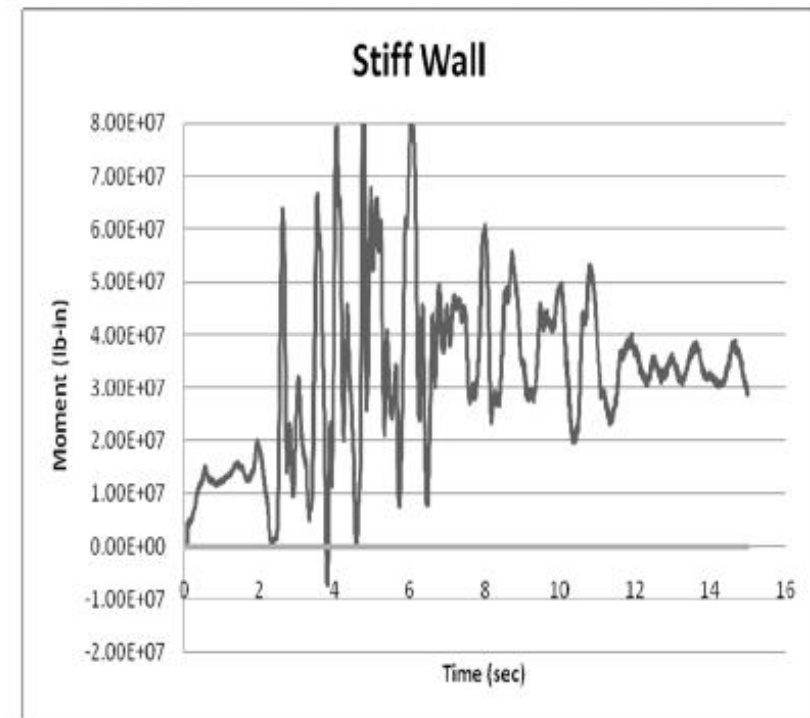
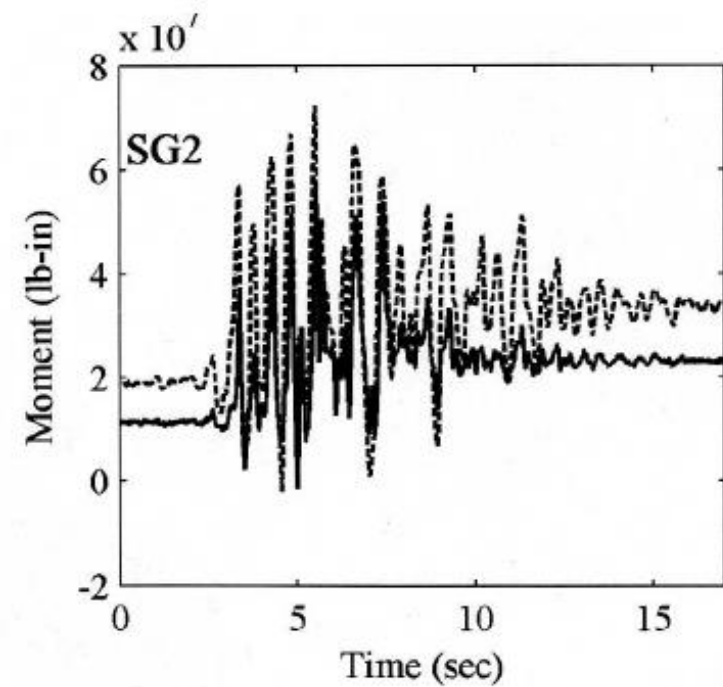


# Finite Element Studies



# Comparisons – Moments at Base of Stiff Wall

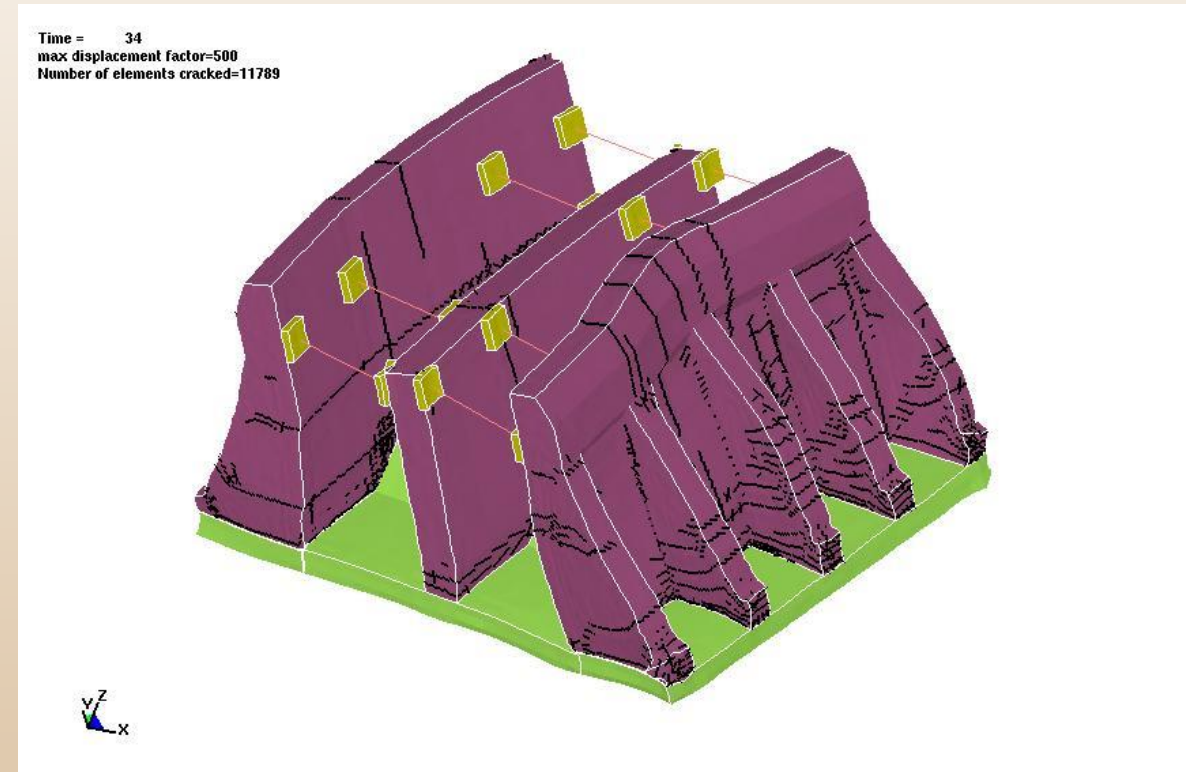
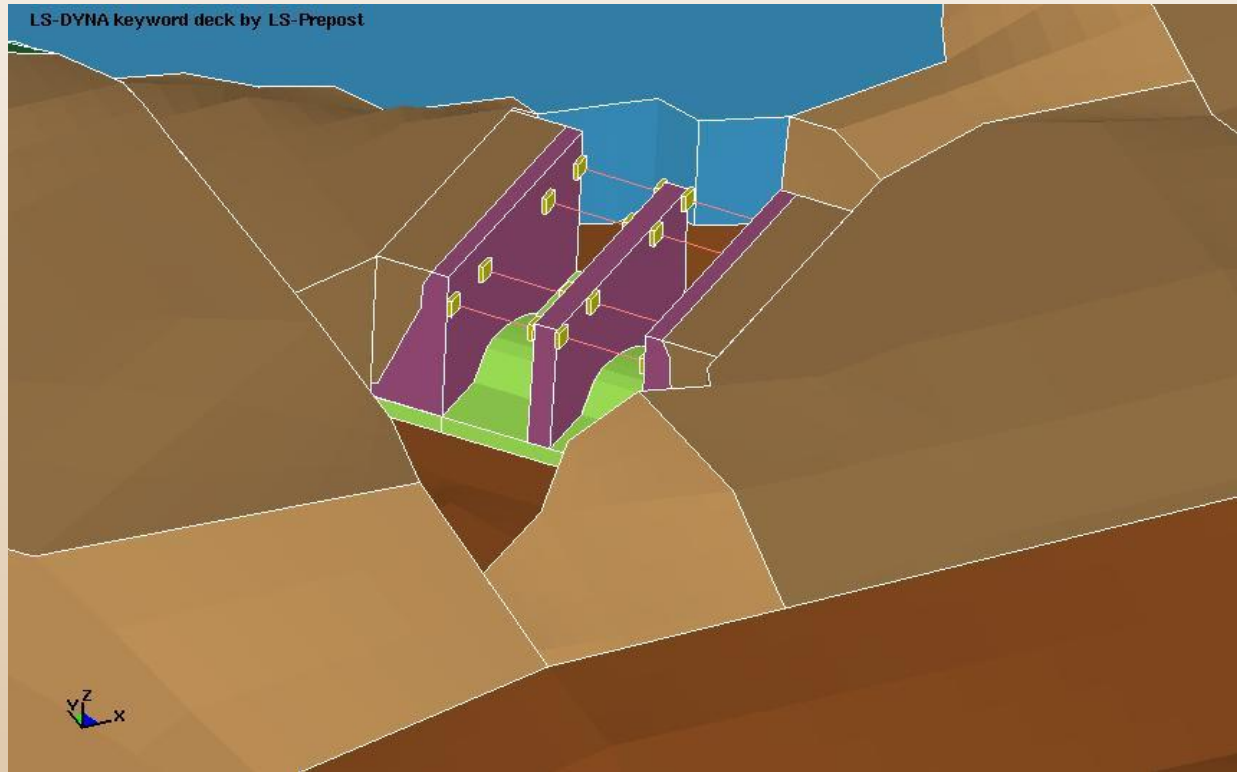
- Top figure
  - Solid line Berkeley centrifuge experimental results
  - Dashed line Berkeley finite element results
- Bottom figure
  - USBR LS-DYNA results using soil material No. 16
- Finite element captures response reasonably well, but slightly conservative.



# Finite Element Model of Embankment-Wall System

- Full embankment dam and foundation included in model – plastic kinematic material models
- Reinforced concrete modeled with non-linear material properties
- Concrete allowed to crack and reinforcement allowed to yield
- Crest structure walls may crack and some of the reinforcement may yield but loads are redistributed and wall may remain stable
- Soil is modeled with non-linear properties so that soil can yield
- Contact surfaces are provided between the wall and the soil backfill
- Significant amount of effort is needed to verify and test model
- Sensitivity analyses are critical to evaluate changes in soil properties and models, boundary conditions and methods of applying loads

# Example Finite Element Model

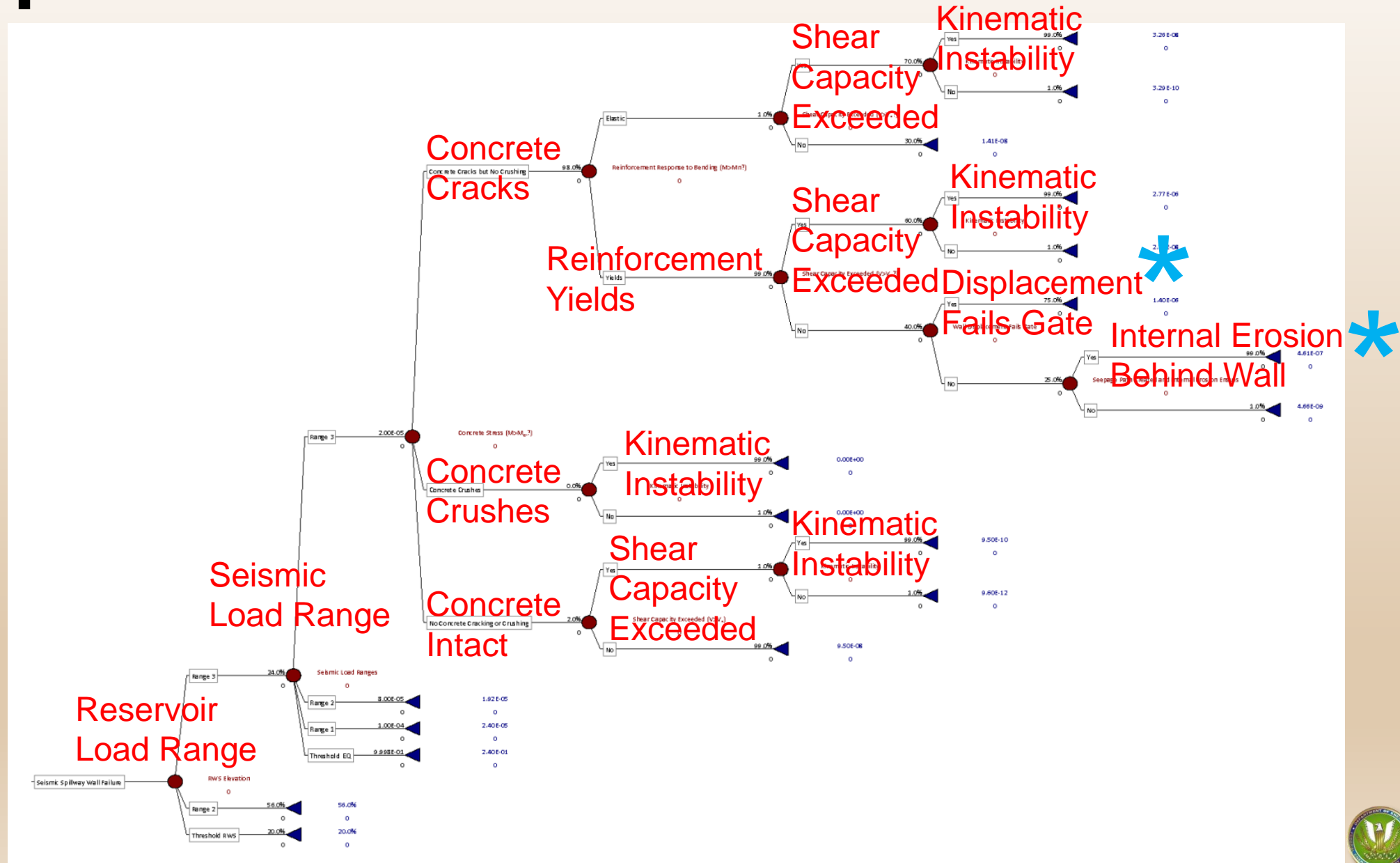




# Finite Element Results vs. Sitar – Al-Atik Research

- While Sitar – Al-Atik research indicates that accepted methods may overstate earth pressure loads on walls, finite element studies of spillway crest structure walls indicates that earth pressures can be greater or less than accepted methods
- The primary differences between the Sitar - Al-Atik results and the FLAC and LS-DYNA finite element results are:
  - Various geometries and backfill conditions for crest structure walls
  - Various foundation conditions
  - Finite element studies have indicated earth pressures approaching a passive condition for spillways adjacent to rock abutments

# Example Event Tree



# Takeaway Points

- Gated spillway walls retaining embankment soils are subject to increased loading during earthquake shaking
- If water is being stored against the gates at the time of an earthquake, potential failure modes and consequences exist
- If the loading causes excess deformation or collapse of such a wall, the adjacent gate could fail and/or a seepage path could open up along the wall through which internal erosion could take place
- An evaluation of the stability of these walls, including seismic earth pressures, is often needed to evaluate the risk posed by these potential failure modes.



# E-7 Seismic Retaining Wall Wall Example

- Consider a spillway with crest structure cantilever walls that are 2-feet thick at the base and 40-feet high. Calculate the shear stresses at the base of the wall for the earthquakes described in Table V-7-2, using Mononobe-Okabe for active earth pressures and assuming cohesionless backfill with a friction angle of  $30^\circ$  and a density of  $120 \text{ lb/ft}^3$ . Assume that the backfill is at the top of the walls and that the backfill surface is horizontal. Assume that the angle of interface friction ( $\delta$ ) is  $15^\circ$ . Assume that the shear capacity of the spillway walls is  $200 \text{ lb/in}^2$ , and that it remains constant for all loading conditions. Based on a comparison of the shear stress at the base of the wall to the shear capacity of the wall concrete, estimate the probability that the shear capacity will be exceeded for the 1000-, 5000-, 10,000-, and 50,000-year earthquake. Assume that there is no vertical component of the ground motions.



# Wall Example

Table V-7-2 – Spillway Wall Analysis – Earthquake Loads	
Recurrence Interval, yr	Peak Horizontal Ground Acceleration
1000	0.1g
5,000	0.2g
10,000	0.3g
50,000	0.4g

# Wall Example Solution

- The active earth pressure coefficients were obtained from the figure for a friction angle of  $30^\circ$ . The active earth pressure coefficients are shown in Table 1. The total earth pressure (both static and dynamic) were calculated using the following equation:
- $P_{AE} = K_{AE}[1/2(\gamma_t(1 - k_v))H^2]$
- The vertical ground acceleration was assumed to be 0. The shear at the base of the wall is calculated below.
- $V = K_{AE}[1/2(\gamma_t)]H^2 = K_{AE}[1/2(120)]40^2$
- The shear stress at the base of the wall is calculated below for the 1000-yr earthquake:
- $v = V/(24 \times 12) = 33,600/288 = 117 \text{ lb/in}^2$
- The following table summarizes the other load case results:



Table 1 - Shear Stresses at Base of Wall				
Recurrence Interval, yr	Peak Horizontal Ground Acceleration	$K_{AE}$	Total Earth Pressure Force	Shear Stress at Wall Base
1000	0.1g	0.35	33,600 lb	117
5000	0.2g	0.43	41,280 lb	143
10,000	0.3g	0.55	52,800 lb	184
50,000	0.4g	0.70	67,200 lb	233

Based on the calculated shear stresses above and the shear capacity of 200 lb/in<sup>2</sup>, the following estimates that the shear capacity will be exceeded were made (see response curve in reinforced concrete section):

1000 yr – 0.001

5000 yr – 0.001

10,000 yr – 0.1

50,000 yr - 0.99